Research Article

Parametric Analysis on Seismic Performance of Hybrid Precast Concrete Beam-Column Joint

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In this paper, a nonlinear finite element (FE) analysis of high-performance hybrid system (HPHS) beam-column connections is presented. The detailed experimental results of the ten half-scale hybrid connections with limited seismic detailing have been discussed in a different paper. However, due to the inherent complexity of HPHS beam-column joints and the unique features of the tested specimens, the experimental study was not comprehensive enough. The new connection (HPHS) detail suggested in this study is characterized by ductile connection, steel connectors, and engineered cementitious composite (ECC) which is a kind of high-performance fiber reinforced cement composite with multiple fine cracks (HPFRCCs). Therefore, in this paper, FE analysis results are compared with experimental results from the cycle tests of the two specimens (RC and PC) to assess model accuracy, and detailed model descriptions are presented, including the determination of stiffness and strength. The critical parameters influencing the joint's behavior are the axial load on column, beam connection steel plate length, inner bolt stress contribution, and plastic hinge area.

1. Introduction

Precast concrete has not been used widely as a framing system for buildings located in several seismic regions. Precast concrete joints between the prefabricated members have some issues. Connections, in particular beam-to-column connections, are the vital part of precast concrete construction. To satisfy the structural requirements of the overall frame, each connection must have the ability to transfer vertical shear, transverse horizontal shear, axial tension and compression, and occasionally bending moment and torsion between one precast component and another, safely. The transfer of forces between the components and eventually the behavior of frames are governed by the characteristics of the connections. However, in practice, the behavior of precast connections is not well established and not fully understood to fulfill the requirements needed in the design and construction development of precast technology [1–3].

Current technology is widely available to satisfy the growing demand required of engineers to provide communities with superior levels of structural performance

during an earthquake. As more advances are made in seismic engineering, the available technology becomes more cost-competitive when compared to traditional construction practice: further financial benefits can be associated with the improved response of the system considering the seismic risk applied over the working life of the structure. High performing systems will be designed to operate more efficiently as they are tuned to their direct application. As a result, the seismic demand imposed onto a structure (maximum displacements and accelerations) can be significantly reduced, thereby reducing material costs and construction time. However, in developing this new technology, design recommendations are required to ensure the technology is appropriately utilized.

This paper focuses on understanding the behavior of the HPHS (high-performance hybrid system) under seismic action. The validity of the HPHC system was demonstrated by a series of experimental tests, which proved the system has good performance under lateral loading. For application of the HPHS to the real structure, the seismic performance of single connections for several primary variables has to be

assessed. In this study, parametric study for the HPHS connection was carried out based on test and FEM analysis. Investigated parameters include (1) axial load on column, (2) beam connection steel plate length, (3) inner bolt stress contribution, and (4) plastic hinge area. Therefore, this paper is aimed at developing and calibrating a nonlinear FE model and further uses it to investigate the behavior of HPHS by varying the main control parameters [4].

2. Previous Development Connection and Test Results

The new connection detail suggested in this study is characterized by ductile connection, steel connectors, and engineered cementitious composite (ECC) which is a kind of high-performance fiber reinforced cement composite with multiple fine cracks (HPFRCCs) and used in order to improve the constructability of joint and efficiently transfer stress between discontinued precast members. Making steel connector consists of bolting steel tubes and steel plates which are usually placed inside the precast column and beam and casting the ECC to some parts of the beam and joint in the field (refer to Figures 1, and 2) [1–3].

The dimensions and details of reinforcements of the specimens are shown in Figure 3, and the other experimental parameters are listed in Tables 1 and 2. Stress discontinuity between the members associated with the steel connectors and the ECC was not observed, and the load was effectively transferred to the beam and joint. The connection detail developed and suggested in this study which has elastic joint and steel plate connector satisfied the requirement prescribed in the ACI structural guideline and thus was verified to provide excellent seismic performance (refer to Figure 4) [1–3].

3. Analytical Model

The proposed connection system (HPHS) has various types of structural elements such as concrete, steel plate, and ECC. It is very difficult to investigate design variable experiment, so nonlinear finite element analysis was conducted based on the experimental results. To improve understanding of the proposed connection system of local stress-strain behavior and joint strength in the vicinity of beam-to-column connection, a total of three concrete models were adopted and analyzed. These innovative high-performance hybrid systems (HPHSs) make use of steel, bolt sections, and ECC into the beam-column joint region to facilitate the connection of precast elements. In the experimental study, two cast-in-place and ten PC specimens, whose connection configurations slightly differed from each other, were tested. However, due to unique features of the tested specimens and material heterogeneity, it was difficult to understand the complex seismic behavior of beam-column connections. Furthermore, the effect of several influencing parameters such as flexural strength ratio and axial load cannot be varied in a limited number of experiments. The ABAQUS (ABAQUS version 6.6-1, 2006) finite element code was used to analyze the proposed precast beam-column connections. These

FIGURE 1: Description of development of precast concrete beam-to-column connection.

numerical models consisted of a combination of elements, springs, and constraint conditions. Amongst these were refined 2D plane stress elements incorporating the full nonlinear material/geometric properties, contact elements, surface interaction with friction, constraint conditions using equation points, concrete crack conditions, and elastic foundation springs. These advanced modeling methods were intended to provide a detailed and accurate understanding of the overall behavior of the connections, including the stress distributions on the contact surfaces in spite of the high computational cost typically associated with this type of data [4, 5].

3.1. Material Properties. The longitudinal reinforcement of the beam and column was deformed bars of yield strength 437 MPa and 508 MPa, while the beam stirrups and column transverse ties were applied with yield strength 475 MPa and 400 MPa. The slump value of the concrete mix was 75 ± 25 mm. The average compressive strength of concrete calculated using the cylinder tests was found to be 27.5 MPa (joint area ECC = 40.5 MPa). Steel connector, used in the construction of the specimens, was confirmed to be SS400. Average values of steel section properties were obtained from the samples of tensile coupon tests. However, for the wide range of parametric study, proposed equations were used and compared with test results. Used stress-strain models of concrete were the modified Kent–Park model and Collins model accounting for the confinement effect of steel tube embedded in the column. Additionally, the concrete damaged plasticity model was provided by ABAQUS; it needs the true stress-logarithmic stress-strain relation for tension and compression. Therefore, in this research, an equivalent uniaxial constitutive model for concrete in tension suggested by Torres [6] was used. It is also shown in Figure 5. The steel material properties for the steel plate and column tube were modeled after SS400 with fully nonlinear isotropic characteristics. For the same reason of concrete, steel was modeled to simplified hardening material as shown in Figures 6 and 7. The stress-strain curve of reinforcements was also determined by the test. However, the flexural moment ratio is an important variable for the design of

FIGURE 2: Proposed connection type. (a) Inside connection. (b) Outside connection.

FIGURE 3: Details of specimens. (a) RC-control. (b) PC-I50-0.2.

TABLE 1: Details of test specimen and material properties.

Specimens	Connection method (axial force)	Hoop bar of joint area	ECC area (mm)	Column size (mm)	Beam size (mm)	Column							
						Reinforcing bar (upper and lower)		Hoop					
						f_{cy} (MPa)	ρ_c	n_c	f_{hy} (MPa)	ρ_h	s_h (mm)	n_b	
RC-control	Cast-in-place (0.1)	O	—										
PC-I50-0.2	Inside connection (0.2)	X	500 (1.4 d)	350 × 350	350 × 400	508	0.038	12-D22	475	0.011	50	D13	
Specimens	Reinforcing bar (upper and lower)		f'_c (MPa)										
	f_{by} (MPa)	ρ_{bu}, ρ_{bl}	n_b	Stirrup		PC member	ECC member	$(\sum M_c / \sum M_b)$	v_{j1} (kN)	v_{j2} (kN)	v_{by}^{by} (kN)	(v_{j1} / v_{by})	(v_{j2} / v_{by})
RC-control	437	0.012	8-D16		27.5	—	—	1.6	1,056	792	629	1.68	1.53
PC-I50-0.2	461	0.016	8-D19		27.5	40.5	40.5	1.6	1,286	965	628	2.04	1.53

f_{cy} , f_{hy} , f_{by} and f_{sy} : yield strength of column bar, hoop, beam bar, and beam stirrup, respectively; ρ_c , ρ_b , ρ_{bu} , ρ_{bl} , and ρ_s : ratio of column bar, hoop, upper beam bar and lower beam bar, and stirrup, respectively; n_c , n_b , n_s , and n_s : size of column bar, hoop, beam bar, and stirrup, respectively; f'_c : compressive strength of concrete and ECC; v_{j1} and v_{j2} : type 1 and type 2 shear strength of joint predicted by ACI 318-05; v_{by} : joint shear strength when beam bar yields.

TABLE 2: Test results.

Specimen		P_y (kN)	P_{max} (kN)	P_f (kN)	δ_y (%)	δ_{max} (%)	δ_f (%)	u [-]	M_n (kN. m)	M_{uj} (kN. m)	M_{peak} (kN. m)	v_n (kN)	v_u (kN)	v_{peak} (kN. m)	(v_{peak}/v_u)
RC-control	Pos	89	119	114	1.5	3.5	4.25	2.3	125	285	206	792	540	572	1.06
	Neg	95	127	120	1.9	3.5	4.25	1.8			219			611	1.13
PC-I50-0.2	Pos	111	148	114	1.3	3.5	4.25	2.7	125	346	255	964	583	711	1.22
	Neg	124	166	110	1.3	3.5	4.25	2.7			286			798	1.36

All estimates associated with moment and shear are computed based on actual material properties. P_y : moment at first yield of top bar (measured); P_{max} : peak load (measured); P_f : failure load (measured); δ_y : yield displacement (measured); δ_{max} : peak displacement (measured); δ_f : failure displacement (measured); u : ductility (δ_{max}/δ_y); M_n : nominal moment; M_{uj} : moment corresponding to v_n (computed); M_{peak} : peak moment (measured); v_n : nominal joint shear capacity (computed); v_u : joint shear demand (computed); v_{peak} : joint shear at M_{peak} .

(a)
FIGURE 4: Continued.

FIGURE 4: Test and analysis results. (a) RC-control. (b) PC-I50-0.2.

FIGURE 5: Stress-strain curve of concrete in uniaxial stress state.

moment frame, and stress-strain relation for other reinforcements would be needed. Therefore, we also use the stress-strain relation for various diameter of reinforcements (refer to Figure 6). Table 3 indicates the material input codes of the concrete damage plasticity for ABAQUS.

The last set of material properties is ECC in compression and tension. As previously discussed, ECC has very large capacity in tension. Using stress-strain data from the test, equivalent perfectly plastic material behavior was used for material modeling in tension (refer to Figure 8). And available test data were used for compression.

3.2. Finite Element Modeling. In the present study, the specimens were analyzed using the ABAQUS software. Two-dimensional (2D) plane stress elements were applied to simulate the concrete and steel plates, while reinforcing bars were modeled as truss elements. In material modeling, the concrete models were based on nonlinear fracture mechanisms to account for cracking, and plasticity models were used for the concrete in compression and steel reinforcement. The ABAQUS (ABAQUS version 6.10-1, 2010) finite element code was used to analyze the effect of variables of the proposed connection system (refer to Figure 9).

Generally, in the composite structure, there are many problems with contact area of concrete and steel. Therefore, all interfaces between two contact surfaces were constrained with each other. The general contact formulation used in ABAQUS involves a master-slave algorithm. This formulation considers the hard contact for normal direction of

FIGURE 6: Stress-strain curve of steel in uniaxial stress state.

FIGURE 7: Stress-strain curve for confined concrete.

TABLE 3: ABAQUS input values for confined concrete.

Plastic stress (MPa)	Plastic strain	Compression damage
28.21	0	0
32.03	0.000110	0.01
35.23	0.000221	0.02
39.80	0.000493	0.05
41.92	0.000834	0.1
41.99	0.001032	0.15
41.29	0.001450	0.2
39.89	0.002287	0.5
21.22	0.01331	0.9

each surface and frictional contact behavior for tangential direction. Surface interactions and constrained area are shown in Figure 10. Boundary conditions for generated model are shown in Figure 11. Bottom of the column was supported by pin, and the end of the beam was supported through roller condition. These boundary conditions were determined from the test assumptions. The beam end was restrained only in roller, and the bottom of the column was restrained with a hinge. The FE models were loaded in two steps. Axial load was applied in first step at the column, and

FIGURE 8: Stress-strain curve for ECC in uniaxial stress state in tension.

FIGURE 9: Partitioned 2D models for proposed connection system (hybrid connection).

lateral load was applied to the same location in second step, cyclically. Cyclic load was applied by displacement control, and this is shown in Figure 12.

3.3. FE Analysis Results. To verify the finite element model, the analytical results were compared with the experimental results. The specimens were modeled with a truss elements and the remaining plane stress 2D elements. Concrete was modeled using 2D plane stress elements which were iso-parametric elements. On the other hand, the reinforcing steel bars were modeled as two-node truss elements. At the joint core region, the area of truss elements close to the boundary was increased appropriately to simulate their corresponding steel area contributions. The beam bottom bars were discontinued at the face of the column. Steel plates, which were used for the connection at the joint, extended inside the beam at one side and abutted with the column face on the other side. These plates were simulated as 2D plane stress elements. These elements were assigned with steel plate thickness and its material properties. The concrete on the front and rear side of these elements was neglected in the analysis as it was filled up after the connections were fastened. Four rows of 2D elements at the bottom of the joint were treated as being connected by the steel plates, and their equivalent area was transferred to the column main bars and transverse links.

FIGURE 10: Constrained model.

FIGURE 11: Boundary condition.

FIGURE 12: Loading sequence.

The predicted and observed responses of the specimens are presented in Figure 13. From Figure 13(a) of specimen RC-control, it can be seen that the analytical model seemed to have predicted a good response with respect to the experimental observations. Although the displacements of the analytical model for a few initial cycles were slightly higher, the later cycles' predicted results were in good agreement with the experimental counterparts. Specimen RC-control achieved a displacement ductility factor (μ) of about 2.8, and pinching was observed in the loops. The loops were thin and quite similar to the experimental results. A large deformation of the joint core was observed at this stage. Figure 13(b) shows the analytical and experimental results comparison

for specimen PC-I50-0.2. From the experimental results, it was seen that the specimen exhibited a large initial displacement for many cycles. The specimen achieved good energy dissipation till μ of approximately 2.8. The global deformed shape of the specimen corresponding to μ of 1.5 is given in Figure 13(b). A moderate deformation of the joint core and upper and lower parts of the column was seen from the figure. Specimen I-ECC25 reached μ of approximately 2.9, slightly lesser when compared to its experimental values. Although the experimental loops showed large initial displacements, their analytical counterparts always depicted steady displacements throughout. This may be due to the fact that the connections might have had some initial gaps in the steel connector, where the nuts and bolts were fastened, which might have slipped after the application of load leading to large initial displacements.

Comparison of the analytical and experimental results of all the specimens showed that the lateral load-displacement hysteresis loops obtained from the FE analyses were quite similar to the experimental observations. Besides, the failure modes and the ultimate ductility capacities correlated well with the experimental results. The FE analyses also showed that results of the deformations and cracking patterns matched well with the experimental observations. From the aforementioned observations and predictions of both the global and local behaviors using the FE analysis, the use of FE modeling techniques can, therefore, be further extended to study the joint performance by varying different parameters.

4. Parametric Studies

4.1. Influence of Axial Loads on Behavior of Beam-Column Joints. Axial loading is a critical parameter in the studies of beam-column joints, but its effect on seismic behavior of beam-column joints has not been fully understood. Previous investigations have shown that axial force is beneficial to the joint shear resistance [3]. Since the neutral axis depth in the column increases with axial compression load, a larger portion of the bond forces from the beam bars can be assumed to be transferred to the diagonal strut. Therefore, the concrete contribution to the joint shear resistance will be increased. In [7], the authors experimentally investigated two nonductile interior beam-column joints with different axial loading levels. However, both of these specimens failed due to the pullout of the embedded beam bottom bars instead of joint shear failure.

Lin and Restrepo's investigations [8] showed that axial compression in excess of $0.3f'_cA$ became detrimental to the joints. In a study conducted by Fu et al. [9], it was pointed out that if the shear was small, the increase of axial loads was favorable to the joints, whereas for high shears, the increase of axial loads was unfavorable. Li et al. [10] found that for a rectangle joint, an axial load less than $0.4f'_cA_g$ was beneficial to the joint, while the axial compression load ranging between zero to $0.2f'_cA_g$ enhanced the joint's performance for deep wall-like column joints.

In this study, the influence of axial loading on the seismic behavior of hybrid-steel concrete joints is investigated. Axial load was applied in first step at the column, and lateral load

(d)

(e)

(f)

FIGURE 13: Predicted story shear forces versus horizontal displacement and crack pattern. (a) RC-control. (b) PC-I50-0.2. (c) PC-50H-0.1. (d) PC-I25H-0.1. (e) PC-O50-H-0.1. (f) PC-O25-H-0.1 (exterior).

was applied to the same location in second step, cyclically. The same loading histories as those used in the analysis of specimens without axial loading were applied, and the story shear force versus horizontal displacement plots corresponding to different axial load levels were plotted for specimen PC-I50-0.2. It can be seen that Specimen inter series attained an optimum value of ultimate story shear when axial load ratio was $(N/A_g f'_c) = 0.2$. Therefore, the analysis results suggested that the axial load $(N/A_g f'_c) \leq 0.3$ was beneficial to the joints' performance. However, the axial load ratio $(N/A_g f'_c) > 0.3$ was found to be detrimental as it reduces the story shear and energy dissipation of the joint (refer to Figure 14).

4.2. Formation of Plastic Hinge. When designing the reinforced concrete frame, the most important design consideration is the moment strength ratio of beam and column and plastic hinge location. Furthermore, steel plates would be a very important design parameter because this parameter determines the strength and stiffness of beams.

The moment strength ratio is defined as the ratio of the sum of the nominal moment strengths of the column above and below the joint to the sum of the nominal moment strengths of all the beams in one plane framing into the joint. Theoretically, a moment strength ratio greater than one should cause the plastic hinges to form in the beams and not in the column. However, ACI318 Building Code [2] and ASCE-ACI Committee 352 recommend the moment strength ratio to be greater than or equal to 1.2 and 1.4, respectively. For the beam failure of proposed connection, specimens were designed to have the moment strength ratio as 1.6. For designing the proposed connection system more economically, plate length and thickness should be examined widely. Therefore, in this section, FE models were investigated. Investigated range of connection detail was determined according to the moment strength ratio of 1 to 1.6.

As a result of finite element analysis, the capacity of beam-to-column joint specimen could be determined by the strength of beam for most cases. However, the failure mode of the beam-to-column specimen for the case of moment capacity ratio 1 had shown beam-joint failure. Other test specimens which have a moment strength capacity of 1.2~1.6 have shown beam failure mode. As the moment capacity ratio increased, the plastic hinge location moved to the outside of the joint. And cracks were more concentrated to plastic hinge. It is shown in Figures 15 and 16.

Strain distribution is very important information for plastic hinge location; from the strain information of FEA results, plastic hinge location was clearly observed. All specimens have experienced yielding of reinforcements. However, decreasing the moment strength ratio decreases the stress in plastic hinge region. At low level of moment strength ratio, yielding of reinforcement was shown latterly, than other cases of specimens. It is shown in Figure 17. Furthermore, to verify the contribution of steel plate and to locate the plastic hinge region, more FE analyses were performed. As described above, the stiffness of the beam

FIGURE 14: Load-displacement predictions under different axial loading levels (specimen PC-I50-0.2).

section is the main parameter for locating plastic hinge region. As a result of FE analysis, the crack pattern of each FE model, which has different lengths of connecting plate, is clearly different. The location of plastic hinge was determined by the length of plates clearly. This is shown in Figure 18.

5. Design Consideration

To determine the moment capacity of bolt connection, the strain distribution of bolts and connecting plates should be investigated. Because bolt strain cannot be measured from the test, 3D finite element analysis was performed. Because too many components were needed to compensate details of the proposed connection system, analysis was performed in monotonic loading, only. The finite element model consisted of 3 parts: concrete parts, steel parts, and reinforcement. For the reality of finite element modeling, each part was modeled separately. And reinforcements were embedded in concrete components. These models are illustrated in Figure 19. Boundary conditions and other restraints are made with the same method of 2D finite element analysis.

As a result of finite element analysis, stress contour and strain field were provided. Especially, stress and strain of bolts components were investigated for the distribution of resultant forces for connecting area. The stress distribution is illustrated in Figure 20. In the connecting area, most of stress was concentrated in end of the plate. Because stress was flowing along the normal direction of contact area, stress concentration of concrete was shown at the end part of the steel plate. However, concrete of the column remained in elastic range. For locating the plastic hinge of beam elements, the stress distribution of reinforcements was investigated. Basically, stress was concentrated at the end of the steel plate. Therefore, using the proposed connection system, plastic hinge could be controlled by the plate length. This is shown in Figure 21. For the design purposes, stress

FIGURE 15: Crack pattern by axial load (specimen PC-I50-0.2). (a) Axial load ratio of 0.1. (b) Axial load ratio of 0.2. (c) Axial load ratio of 0.3. (d) Axial load ratio of 0.5.

FIGURE 16: Crack pattern according to moment strength ratio (specimen PC-I50-0.2). (a) Moment strength ratio of 1.0. (b) Moment strength ratio of 1.2. (c) Moment strength ratio of 1.4. (d) Moment strength ratio of 1.6.

FIGURE 17: Stress distribution according to moment strength ratio (specimen PC-I50-0.2). (a) Moment strength ratio of 1.0. (b) Moment strength ratio of 1.2. (c) Moment strength ratio of 1.4. (d) Moment strength ratio of 1.6.

FIGURE 18: Plastic hinge location according to plate length. (a) Tested length. (b) Half of tested length.

FIGURE 19: Finite element modeling (embedded components). (a) Concrete part. (b) Steel plate part. (c) Reinforcement part. (d) Total modeling.

FIGURE 20: Stress distribution of total model.

FIGURE 21: Stress distribution of reinforcements.

FIGURE 22: Stress distribution. (a) Steel plates. (b) Connection bolts.

TABLE 4: Stress distribution and resultant force results.

Element ID	Stress type	Strain compatibility		FEM	
		Stress (MPa)	Resultant force (kN)	Stress (MPa)	Resultant force (kN)
El. 1	Tension	400	624	368	574
El. 2	Tension	280	302	253	272
El. 3	Tension	180	151	150	125
El. 4	Tension	90	76	65	54
El. 5	Compression	92	77	113	94
El. 6	Compression	200	73	253	92
Cc	Compression	30	770	36	720

distribution of connecting steel plate would be needed. Using the information of stress distribution at the center of bolt element, which was described in this section, resultant force could be calculated. This is also shown in Figure 22.

Neutral axis was formed at the third bolt from the compressive extreme fiber. Because stiffness could be increased with the composite action at compressive area, compressive stress area was bigger than tensile one. Provided stress is shown in Table 4.

6. Conclusion

FE models used in this study well predict the behavior of test specimens. However, the initial stiffness problems encountered by the embedded truss model should be solved for the more wide parametric study such as serviceability check in design process.

According to the parametric study, the effect of axial load was investigated. There is a reduction in story shear and ultimate number of cycles after enhancement in axial load ratio beyond 0.3 (i.e., $(N/A_g f'_c) > 0.3$). Therefore, the analysis results suggested that the axial load $(N/A_g f'_c) > 0.3$

was beneficial to the joint's performance. However, the axial load ratio $(N/A_g f'_c) > 0.3$ was found to be detrimental as it reduces the story shear and energy dissipation of the joint.

Plastic hinge region is very important design criteria for frame action. From the result of the parametric study, plastic hinge location is determined by the moment strength ratio and plate length. Generally, plastic hinge was formed far from the column face with the increase of moment strength ratio. It was clarified by investigating the stress distribution of reinforcement. For low value of moment strength ratio, reinforcement is hard to yield and plastic hinge cannot be formed easily, in the area of plate installed area. However, the length of plates is an important parameter for location of plastic hinge. Investigation from the test and FE model revealed that plastic hinge occurred at the end of steel plates. Therefore, using the proposed connecting system, plastic hinge location can be controlled by the designers.

Complicated 3D model for the proposed connecting system would be used for design strength calculation for connection. The stress distribution of bolt elements is similar to the stress distribution resulted from strain compatibility

methods. Therefore, the proposed connection can be designed using conventional design methods.

Data Availability

The data used to support the findings of this study are included within the article. Any additional data related to the paper may be requested from the corresponding author.

Conflicts of Interest

The author declares that there are no conflicts of interest.

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